

Preliminary Stormwater Management Report

Matheson & Rosedale Subdivision

EFI Engineering Inc.

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1 BACKGROUND

Monument Group (“Monument”) was retained by EFI Engineering Inc. to prepare a Preliminary Stormwater Management Report for the proposed Matheson & Rosedale Subdivision. The 23.54ha site is located in the Township of Montague situated in Lanark County and the jurisdiction of Rideau Valley Conservation Authority (RVCA). The subject property has frontage both on Matheson Drive and Rosedale Road South zoned as Rural Residential. Figure 1 illustrates the site location.



Figure 1-1: Proposed Development Location

The site is generally bound by rural residential properties fronting Matheson Drive and Rosedale Rd S. Various pastures make up the subject property with some small, treed area in the northeast corner. The primary usage of the fields is pasture and wheat harvesting.

The topography of the land slopes towards the adjacent roadways in generally a west to southwest direction. The majority of property drains towards Matheson Drive with relatively low slopes of 0.5% - 1%, while the remainder of the property slopes southwest 3 – 4% towards Rosedale Road S.

The closest watercourse is the main channel of Rosedale Creek which intersects properties south of Rosedale Rd S. The creek is a tributary to the Rideau River System. No other regulated features (i.e., wetland, watercourses) are located within or immediately adjacent to the property.

1.1 Proposed Development

The proposed development will consist of 41 detached single-family homes on 1 acre lots with a minimum frontage of 46m (150ft). Potable water will be drawn at each lot from private wells and disposed of in individual septic systems. The roadway will be a typical rural right-of-way, 20m wide, with a 7m driving surface, and 1.0m wide bottom ditches to convey runoff from the site to a stormwater management (SWM) facility.

1.2 Background Studies & Plans

The following reports in relation to the development were referenced here within:

- *Issued for Draft Plan Approval Drawing Set*, prepared by EFI Engineering, November 22, 2024.
- *Rosedale Subdivision, East of Smiths Falls, ON Geotechnical Subsurface Investigation Report No. 23C258*, prepared by St. Lawrence Testing & Inspection Co. Ltd., October 31, 2023.
- *Hydrogeological Assessment Report – Matheson and Rosedale Subdivision, Part Lot 20 Concession*, Cambium Inc., July 10, 2024.

2 DESIGN OBJECTIVES

The SWM design was prepared to meet the following objectives:

- 1) **Quantity Control** – The objective is to ensure that post-development peak flows do not exceed the pre-development levels for all storm events up to the 100-yr return period. This ensures that there are no negative impacts due to flooding on any lands downstream of the subject property.
- 2) **Quality Control** – Quality control target will be to meet a Level 1 Total Suspended Solids (TSS) removal efficiency at the *Enhanced* level of 80% long-term TSS removal rate, in accordance with the *MOE SWM Planning and Design Manual (2003)*.

Following Draft Plan approval, objectives 3) and 4) should be detailed further according to the detailed design of the development.

- 3) **Sediment and Erosion Control Measures** – Prepare a sediment and erosion control plan to control and mitigate release of sediment throughout the construction stage.
- 4) **Operation and Maintenance Plan** – Recommend an operation and maintenance plan for the proposed SWM devices to be implemented by the Municipality.

3 HYDROLOGY

Monument used PCSWMM version 7.4.1. to model the pre- and post- development conditions of the site. PCSWMM is a powerful modeling platform that provides various hydrologic and hydraulic modelling capabilities. The user can analyze several SWM components such as stormwater and watershed modelling. Rainfall/runoff modelling was used to determine pre- and post-development peak flows and pond storage requirements.

Three models were created; a “Pre-Development” model, “Post-Development Uncontrolled” model, and a “Post-Development Controlled” model to simulate peak runoff rates for all return period events. Catchment areas are delineated based on pre- and post-development drainage patterns. These catchments are then modelled in PCSWMM using the following parameters:

- Area (ha) – Total area of each catchment
- Width (m) – Width of overland flow path; this is automatically determined based on the flow length
- Length (m) – Longest flow path of overland sheet flow
- Slope (%) – Average surface slope; Monument determines the average slope using the 85/10 Method
- Imperv. (%) – Percent of Impervious area
- N Imperv. (unitless) – Manning’s coefficient for impervious area
- N Perv. (unitless) – Manning’s coefficient for pervious area
- Dstore Imperv. (mm) – Depth of depression storage on impervious area; Monument selected 1mm
- Dstore Perv. (mm) – Depth of depression storage on pervious area; Monument selected 5mm
- Zero Imperv (%) – Percent of impervious area with no depression storage; Monument selected 25%
- Curve Number (unitless) – SCS runoff curve number
- Runoff Coefficient (unitless) – used in the Nash IUH method for calculating time of concentration in PCSWMM using the Airport method

A Hydrologic Methodology Section in **Appendix A** provides a description of the hydrologic inputs and parameters used within the PCSWMM models.

3.1 Precipitation Data

Precipitation data was derived for the area using the Ministry of Transportation (MTO) online IDF lookup curve. An excerpt of this data is presented below.

MTO IDF Curve (IDF Data from 2010)
 Matheson Drive @ County Road 23

IDF Parameters from MTO IDF Curve Lookup						
	2-yr	5-yr	10-yr	25-yr	50-yr	100-yr
Coefficient (A)	20.3	27	31.4	36.9	41	45.1
Exponent (B)	-0.699	-0.699	-0.699	-0.699	-0.699	-0.699

Uses Equation: Intensity = $A \cdot (T_c)^B$

Rain Fall Depths from Table 2a						
Duration	2-yr	5-yr	10-yr	25-yr	50-yr	100-yr
5 mins	9.6	12.8	14.9	17.5	19.4	21.3
10 mins	11.8	15.7	18.3	21.5	23.9	26.3
15 mins	13.4	17.8	20.7	24.3	27	29.7
30 mins	16.5	21.9	25.5	30	33.3	36.6
60 mins	20.3	27	31.4	36.9	41	45.1
2 hrs	25	33.3	38.7	45.5	50.5	55.6
6 hrs	34.8	46.3	53.8	63.3	70.3	77.3
12 hrs	42.9	57	66.3	78	86.6	95.3
24 hrs	52.8	70.3	81.7	96	106.7	117.4

Figure 3-1: IDF Lookup Curve Precipitation Data

The precipitation data was simulated within each model using the rainfall distribution and durations recommended in the City of Ottawa’s October 2012 Storm Design Guidelines identified in **Table 3-1**.

Table 3-1: Rainstorm Distributions & Timesteps

Duration	Timestep
SCS Type II	
24hr	1 hour
12hr	30 minutes
Chicago	
24hr	1 hour
6hr	15 minutes
3hr	5 minutes
AES 30%	
12hr	1 hour

3.2 Pre-development Conditions

Catchment areas were delineated in the “pre-development” conditions to determine outlet locations. Four (4) onsite catchment areas were delineated draining to three separate outlet locations. The location of these outlets and their contributing catchment areas are illustrated in the Pre-development Catchment Area (ST1) drawing in **Appendix B**.

OF1 – is a 600mm diameter corrugated steel pipe (CSP) cross culvert under Rosedale Road S. adjacent to the driveway entrance 876. Thereafter, a small ditch along the north property boundary of house 877 outlets directly into Rosedale Creek. Catchment Areas EX-3 and EX-4 onsite and EXT-1 and EXT-2 offsite contribute to this culvert crossing.

OF2 – Is a 500mm diameter CSP cross culvert under Rosedale Road S. near the existing farm entrance at the south portion of the subject property. Similar to OF1 the cross-culvert outlets downstream to a small ditch extending between property boundaries of house 795 and 805 to outlet directly into Rosedale Creek.

OF3 – Is the south roadside ditch of Matheson Drive located just north of house 969. The contributing area to this outlet is 15.67ha delineated as Catchment EX-1.

Currently, there is no confirmed outlet for this roadside ditch as it is considered a “sag” between two high points near the driveway entrances of house 969 (south) and 902 (north). There is also no evidence of a culvert-crossing, but it is believed to be buried due to lack of maintenance overtime. Therefore, it is believed that runoff in major storms will spill from the ditch in two different directions as described below.

Based on Ontario Land Information LiDAR data:

Spill #1 – is across Matheson Drive, north into the adjacent field and regulated wetland behind house 902 and 886. Thereafter, runoff outlets into a Tributary to Rosedale Creek near the intersection of Wood Road.

Spill #2 – continues down the roadside ditch south of Matheson Drive across the driveway of 969 Matheson Drive to OF1.

Based on these observations, the roadway creates a major ponding scenario that stores runoff within the subject property prior to spilling.

To accurately represent this condition in PCSWMM, a junction (J100) at the upstream side of the ditch was created containing three outlets as described below:

Spill #1 - A 50m long broad-crested weir – Matheson Drive Roadway – Modelled as a trapezoidal shaped conduit at an elevation of 121.41m

Spill #2 - A 5m long broad-crested weir – House 969 Driveway – Modelled as a trapezoidal shaped conduit at an invert elevation of 121.41m.

Also applied to J100 is a Poned Area of 17,500m². This half of the ponding surface area measured within the site at an invert elevation of 120.60m. This allows the system to account for storage upstream of this node (i.e., roadside ditch) and released through the three connected conduits as capacity becomes available. Therefore, no “flooded water” leaves the system throughout the duration of the simulation.

The model was simulated using the storm distributions and timesteps described in the Precipitation Section above for the 100-yr return period event. Results of this simulation at each outfall are provided in **Table 3-2**.

Table 3-2: 100-yr Pre-development Peak Flows

Duration	OF1	OF2	OF3
SCS Type II (m ³ /s)			
24hr	0.729	0.180	0.701
12hr	0.376	0.107	0.402
Chicago (m ³ /s)			
24hr	0.606	0.154	0.593
6hr	0.441	0.124	0.461
3hr	0.351	0.106	0.395
AES – 30% (m ³ /s)			
12hr	0.298	0.082	0.362

3.3 Post-Development Conditions

A “Post-development Uncontrolled” model was created to evaluate the peak runoff to each outfall in the post-development condition without any quantity controls in place. This then determines which outfalls require these measures in order to satisfy the design objectives.

The Post-development Catchment Area Drawing is provided in **Appendix B** and illustrates the drainage patterns that reflect proposed grading for the development. The model does not include storage nodes at either outlet location; however, it is directed to the respective outlet

locations. Majority of the development will drain to the west side of the site through the developers privately owned land at house 987 and into the south ditch of Matheson Drive S. This main outlet location was selected based on the following conditions:

- 1) Matheson Drive ditch south of House 969 offers a sufficient outlet to municipal right of way ditch directly to Rosedale Creek.
- 2) Lowest elevation of the subject property that will allow for the greatest area captured in the post-development conditions that will minimize earthworks.

The imperviousness of the site was calculated to reflect the increase in hardened surfaces (i.e. roofs, roads and driveways). Based on the concept plan, the average house size is 240m² (2580sq.ft) including the garage. An impervious area buffer was also added to consider larger house footprints and outbuildings such as detached garages and sheds that may be considered at full build out. This impervious buffer was calculated by adding an additional 95m² (~1000sq.ft) to the percentage house in each catchment. Overall, the total imperviousness was calculated to be 14% for the total development. **Table 3-3** shows a comparison of the pre-development peak flows and post-uncontrolled peak flows for all the storm distributions under the 100-yr return period event. Hydrologic values used as inputs for the catchments in the “Post-development” models are provided in **Appendix C**.

Table 3-3: 100-yr Peak flows for the Pre- and Post-Development Uncontrolled Conditions

Duration	OF1 (m ³ /s)		OF2 (m ³ /s)		OF3 (m ³ /s)	
	Pre	Post Uncontrolled	Pre	Post Uncontrolled	Pre	Post Uncontrolled
SCS Type II						
24hr	0.729	1.324	0.244	0.142	0.701	0.137
12hr	0.376	0.988	0.146	0.117	0.402	0.073
Chicago						
24hr	0.606	1.273	0.209	0.124	0.593	0.112
6hr	0.441	1.064	0.169	0.156	0.461	0.088
3hr	0.351	0.890	0.144	0.133	0.395	0.090
AES 30%						
12hr	0.298	0.796	0.111	0.051	0.362	0.048

As illustrated in **Table 3-3**, the uncontrolled peak flows at OF2 and OF3 are less than the pre-development flows. This is a result of the contributing area being reduced in the proposed conditions. Therefore, quantity control measures are not required for these outfalls. However, peak flows for OF1 are greater than the pre and **therefore, quantity control is required**.

As identified above, evidence of a culvert was not found at OF-3. In the post-development conditions, very little runoff will now drain to this roadside ditch well under what was assumed to pond and overflow the roadway. To minimize ponding within the ditch, a new culvert crossing should be constructed to convey runoff north under Matheson Drive. A 400mm CSP pipe was modelled in post-development condition, which is sufficient to convey the 100-yr runoff from Catchment PR2.

To determine the storage requirements at OF1, a storage node was added to the post-development model to formulate the “Post-development Controlled” Model. Iteratively, the storage node was tested to determine the 100-yr storm that generated the greatest storage requirement. **Table 3-4** below identifies the storage requirements determined for each storm.

Table 3-4: Storage Requirements for each Return Period Event

Duration	Storage (m ³)
SCS Type II	
24hr	6870
12hr	4672
Chicago	
24hr	6171
6hr	5312
3hr	4217
AES 30%	
12hr	6387

Based on the results, it was determined that the SCS Type II 24hr storm requires the greatest volume of storage. **Therefore, the 24hr SCS Type II storm distribution was selected as the design storm for sizing the quantity control facility.**

3.4 Target Peak Flows

The 2-100yr SCS Type II storm was run in the pre-development PCSWMM model to determine the controlled target flows at OF1 for post-development conditions as illustrated in **Table 3-5**.

Table 3-5: Pre-Development Flows at OF1 For 2-100yr Return Period

Duration	OF1 (m ³ /s)
2-yr	0.121
5-yr	0.250
10-yr	0.351
25-yr	0.492
50-yr	0.606
100-yr	0.729

4 STORMWATER MANAGEMENT

4.1 Quantity Control

An extended detention wet pond is proposed to meet both the quality and quantity control objective as outlined in **Section 2.0**. The basin will be in the west corner of the site as illustrated on the Conceptual SWM facility provided in **Appendix D**.

4.1.1 Detention Basin Design

The facility has been designed to have two cells – a forebay and main cell. A permanent pool is incorporated to provide quality treatment and limit the need for slope through the facility. The elevation of the facility has been set below the inlet of the ditches from the roadway to ensure active storage does not occupy the roadside ditches in the event of the 100-yr storm. The proposed basin has the following design specifications:

- 1.0m Permanent Pool depth
- 0.90m Active storage (inc. extended detention)
- 0.30m freeboard
- 3H:1V side slopes from bottom of pond to permanent pool
- 4H:1V side slopes from permanent pool to top of berm
- Forebay Berm
- Two stage outlet structure
- Emergency spillway
- 3.0m maintenance path
- 4 length to 1 width ratio
- 6m minimum bottom width

Elevations within the facility are as follows:

Bottom Pond Elevation	= 118.15 m
Top Permanent Pool	= 119.15 m
Top Active Storage	= 120.45 m
Top of Berm	= 120.75 m

The above design will provide a total active storage volume of 7656m³ from the bottom of the pond to freeboard (0.30 m below top of berm). The storage node in the Post-development Controlled Model was supplied with a stage-discharge curve which was extracted from 0.10m intervals from Civil3D based on the conceptual layout in **Appendix D**. These incremental areas

were also used to develop the stage-storage-discharge relationship provided in **Appendix E**. The stage-discharge relationship is then supplied to PCSWMM and assigned to an outlet curve from the storage node to the outlet channel downstream.

4.1.2 Outlet Structure

A 1500mm concrete maintenance hole is proposed as the main outlet structure. The inlet to the structure will be a 675mm diameter concrete pipe. Within this structure a concrete wall will contain two control outlets. Outlet 1 is a 150mm diameter orifice set at the invert of the permanent pool of 119.15m. Outlet 2 a 400mm wide concrete broad-crested weir set to an invert elevation of 119.85m.

An emergency spillway will also be constructed in the top of the berm set to the active storage elevation of 120.45m. The spillway is trapezoidal in shape with a 5000mm bottom width, 3H:1V side slopes and 300mm depth. The purpose of the spillway is to act an emergency conveyance route to ensure the banks are not washed out in the case Outlets 1 and 2 become blocked or storm events greater than the 100-yr occur. The spillway is designed to safely control the greater of the 100-yr return period event peak flow.

4.1.3 Overview of Detention Facility

Table 4-1 provides a hydraulic overview of the SWM facility under each return period event. The storage volumes show that each storm event is less than the active storage of 7656m³ provided. The water surface elevations also illustrate that the 2-yr storm event will be conveyed through the orifice structure, while the 5-100-yr events will crest over the weir invert of 119.85m utilizing both outlets.

Table 4-1: Detention Facility Overview Under Each Return Period Event

Duration	Inflow (m ³ /s)	Pond Outflow (m ³ /s)	Depth (m)	WSELS (m)	Storage (m ³)
2-yr	0.287	0.032	0.49	119.64	2494
5-yr	0.563	0.053	0.76	119.91	4055
10-yr	0.778	0.097	0.87	120.02	4754
25-yr	1.071	0.168	1.00	120.15	5577
50-yr	1.300	0.232	1.09	120.24	6222
100-yr	1.533	0.304	1.19	120.34	6870

WSEL stands for water surface elevation.

As mentioned in the Conveyance Section below, the pond outlets into a grassed swale and into the roadside ditch of Matheson Drive down to OF#1. Based on the pond outflow and runoff from the external catchment areas EXT-1 & EXT-2, the post-development peak flows compared to the pre-development peak flows are presented in **Table 4-2**. This also illustrates the success of the SWM facility controlling peak flows to ensure the target release rates are met at OF1. **Therefore, the quantity control objective has been satisfied.**

Table 4-2: Comparison of Controlled Peak Flows to Pre-Development Peak Flows

Duration	OF1 (m ³ /s)	
	Pre	Post-controlled
2-yr	0.121	0.080
5-yr	0.250	0.175
10-yr	0.351	0.265
25-yr	0.492	0.387
50-yr	0.606	0.485
100-yr	0.729	0.587

4.2 Quality Control

As outlined in the MOE *Stormwater Management, Planning and Design Manual (2003)*, (here after referred too: “*Design Manual*”) SWM ponds are suitable quality treatment facilities for drainage areas of 10ha or larger. The proposed development that will be serviced by the SWM pond is approximately 18.75ha in size and will meet the *Enhanced* quality target of 80% long term TSS removal. This includes post-development catchment areas ST1 – ST6. The remaining catchments PR1 – PR4, will be treated using the low impact development (LID) devices such as vegetated filter strips combined with enhanced grass swales.

4.2.1 Quality Storage

Table 3.2 of the *Design Manual* provides permanent pool and extended detention volume requirements for wet ponds serving sites with varying imperviousness levels. Values less than the prescribed imperviousness (i.e., < 35%) can be extrapolated to determine the necessary storage volumes. Based on the Table, a site having an imperviousness of 14% will require 70m³/ha of water quality storage. From this value, the extended detention shall make up 40m³/ha. The MOE minimum storage requirements and provided storage volumes for the SWM pond are Provided in **Table 4-3**.

Table 4-3: Wet Pond Storage Requirements for Quality Control

Pond Feature	MOE Requirement (m ³)	Provided (m ³)
Permanent Pool	562	3750
Extended Detention	750	1982
Total Storage	1312	5732

4.2.2 Erosion Control

Increased runoff from changes in land use may result in greater sediment loading to watercourses due to erosion. SWM ponds help sustain a stable fluvial system downstream by reducing the velocity of outflows. Erosion control is applied according to the *Design Manual* requirement of storing the 25mm 4-hour Chicago “quality storm” and extending its detention period to over a 24-hour period. At the Pond’s outlet, an orifice attenuates inflow from the 25mm event in order to achieve the detention time within the allotted extended detention volume. The quality storm event was modelled to determine the drawdown time of the pond.

Figure 4-1 below illustrates the drawdown time of the quality event from PCSWMM.

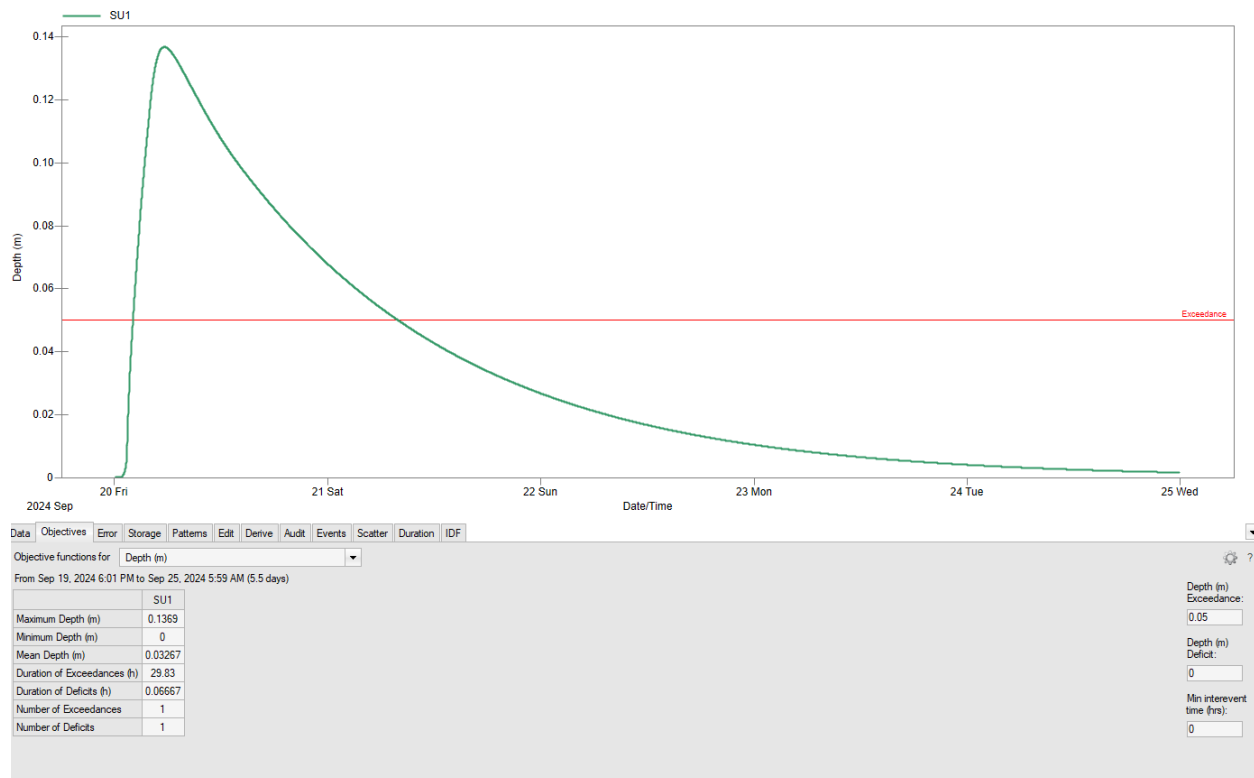


Figure 4-1: Quality Event Drawdown Time

The 25mm event requires a volume of 670m³ which can be stored within the extended detention of 1982m³. In addition, after 30 hours the extended detention is drained to a depth of 0.05m at which point the orifice is no longer submerged and the pond is effectively drained. This meets the MOE requirement of a 24-hour minimum drawdown time.

4.2.3 Forebay Sizing

The purpose of a wet pond forebay is to trap larger particles near the inlet of the pond to reduce the frequency of sediment removal within the main cell. The forebay should be deep enough to minimize resuspension of settled material and long enough to ensure that sediment will settle before entering the main body of the pond. The proposed pond forebay has a depth of 1.0m and a length of ~60m. Several forebay design guidelines have been established by the MOE which are outlined as follows.

Forebay length should at a minimum be long enough to facilitate settling of particles. The minimum forebay length to facilitate settling is 40.66m and was calculated using Equation 1:

(Eqn. 1)

$$Distance_{settle} = \sqrt{\frac{rQ_p}{V_s}}$$

where:

r = length-to-width ratio of forebay

Q_p = peak flow rate from the pond during design quality storm (m³/s)

V_s = settling velocity of particles (m/s)

The forebay should also provide adequate distance to slow the discharge before it enters the main body of the pond. The minimum length of dispersion required to dissipate flows from 100-yr inflow is 24.54m and was calculated using Equation 2:

(Eqn. 2)

$$Distance_{dispersion} = \frac{8Q}{dV_f}$$

where:

Q = inlet flow rate (m³/s)

d = depth of permanent pool in the forebay (m)

V_f = desired velocity in the forebay (m/s)

**The maximum desired velocity in the forebay recommended by the MOE is 0.5m/s*

The *Design Manual* also provides a target average velocity of 0.15m/s across the entire forebay cross-section to reduce the potential for erosion. The average velocity calculated was 0.15m/s which was calculated using Equation 3:

(Eqn. 3)

$$V_{avg} = \frac{Q}{A}$$

where:

V_{avg} = average velocity across forebay

Q = maximum pipe discharge

A = cross-sectional area of forebay

The forebay design provides adequate room for particle settling while keeping anticipated flow velocity within recommended limits. As such, there is little risk for particle resuspension. **Table 4-4** provides a comparison of required MOE forebay characteristics to the design configuration of the forebay.

Table 4-4: Forebay Sizing Requirements and Provided

Forebay Parameter	Required	Provided
Depth (m)	1.0 < depth > 1.5m	1.0
Settling Length (m)	>40.7	60
Dispersion Length (m)	>24.5	60
Width (m)	>5.1	6
Length: Width Ratio	Minimum 2:1	4:1
Average Velocity (m/s)	< 0.15	0.15

The Design Manual also provides a recommended permanent pool area and volume for the forebay to optimize treatment. The maximum criteria for the forebay area is 33% of the total permanent pool over the total wetted area and a preferred volume of 20%. As per the proposed pond configuration, the forebay area accounts for 30% corresponding 27% volume of the entire pond. As the maximum forebay area criteria is not exceeded, the SWM facility meets these design standards as well.

4.2.4 Vegetated Filter Strip

Vegetated filter strips are gently sloping vegetated areas that treat runoff from adjacent impervious areas in the form of sheet flow. These areas reduce runoff velocities and infiltrate runoff into the underlying soils. Filter strips also provide quality treatment by filtering out sediment and other pollutants such as chloride and sodium from de-icing salts. Vegetated filter

strips are ideal as pre-treatment to other LID practices and will be located adjacent to driveways at the lot level and alongside slopes of roadside swales to provide pre-treatment. Design of the vegetated filter strips will include the following design guidance for optimal TSS removal efficiency:

- A maximum flow path length across adjacent impervious surfaces of 25m.
- A minimum flow path length across the filter strip of 5m.
- A slope of 1 to 5%.

Based on available performance studies, pollutant removal efficiencies of vegetated filter strips are highly variable with reported removal efficiencies between 20 to 80% (CVC LID SWM Planning and Design Guide) and therefore should be used in combination with other treatment technologies. Runoff from the vegetated filter strips will be captured in roadside grass swales for secondary treatment.

4.2.5 Enhanced Grass Swales

Grass swales are vegetated open channels that convey and treat stormwater runoff by filtration through vegetation and infiltration through the underlying native soils. Water quality benefits can be enhanced by design features such as modified geometry, check dams, and vegetation.

Grass swales are proposed along the roads to provide quality treatment as well as for conveyance. The Credit Valley Conservation (CVC) 2010 Low Impact Development Stormwater Management Planning and Design Guideline offers the following design guidance for grass swales:

- Bottom width between 0.75 and 3.0m
- Maximum side slopes of 2.5H:1V
- *Maximum velocity of 1m/s for the 4hr 25mm Chicago (Quality) storm
- Length when treating road runoff should be equal to or greater than the contributing roadway length.
- **Pre-treatment with vegetated filter strips

Notes:

*The CVC SWM Planning Design Guideline states that under the desired conditions, an enhanced grass swale's TSS removal efficiency is greater when the swale maintains a velocity target of 0.50m/s under the quality storm event.

**Pre-treatment for side slope between the edge of pavement and toe of swale are accredited as pre-treatment for road surfaces.

As illustrated on the typical road cross-section, the proposed grassed swales will have a bottom width of 1.0m, a channel depth of 0.66m and side slopes of 3:1.

The PCSWMM “Post-Development” model was simulated under the quality event as mentioned under **Section 4.2.2**. The largest peak runoff was calculated from Catchment PR2 of 0.02m³/s. Manning’s Open Channel Flow Equation was manipulated to determine the corresponding velocity and flow depth. Based on a maximum longitudinal slope of 2% and ditch configuration for the typical road cross section, the channel flow depth would be 0.04m with a velocity of 0.446m/s. Therefore, a TSS Removal efficiency of 70% can be assigned, which is less than the 76% TSS removal efficiency recorded for these swale types in optimum configurations.

4.2.6 Total TSS Removal

The total volume of precipitation to be treated is determined by applying the 25mm, 4hr storm to the area of driveways and roadways within each catchment. Runoff from rooftops is considered clean and does not require treatment given it is discharged from roof leaders to the pervious surfaces. PR4 also does not require quality treatment as this is the backslope of the pond berm only.

TSS removal efficiencies are based on median removal rates from available performance studies provided in the CVC LID SWM Planning and Design Guide. The proposed treatment train approach for runoff from driveways and roadways is summarised in **Table 4-5** below.

Table 4-5: Treatment Train Approach for Driveways & Roadways

Quality Treatment Train			
Catchment	Treatment Type	LID	Assumed TSS Removal Rate
PR1 – PR3	Pre-treatment (Roadway)	Vegetated Filter Strip	30%
PR1 – PR3	Pre-treatment (Driveway)	Vegetated Filter Strip	50%
PR1 – PR3	Final treatment	Enhanced Grass Swale	70%

Note: TSS removal rates of vegetated filter strips treating roadways are assumed to be less than for driveways because filter strips treating roadways are located between the edge of pavement and toe of the roadside grass swales and thus, they have a comparably shorter flow path and greater slope. Therefore, to be conservative a 30% removal efficiency is assigned for vegetated filter strips.

The quality treatment matrix below, identifies the performance of the LIDs each catchment. As can be seen, the TSS removal rate provided by each LID is applied to the volume of remaining “untreated” runoff after each treatment device.

Volume Untreated		Volume Treated	
PR1 & PR3			
56m ³ Precipitation (2,260 m ² x 0.025m)			
Driveway (669m ²)		Roadway (1591m ²)	
16m ³		40m ³	
Vegetated Buffer Strip 50% TSS Removal		Vegetated Filter Strip 30% TSS Removal	
8m ³	8m ³	12m ³	28m ³
Enhanced Grass Swale 70% TSS Removal		Enhanced Grass Swale 70% TSS Removal	
2m ³	14m ³	32m ³	8m ³
PR2			
32m ³ Precipitation (1,288m ² x 0.025m)			
Driveway (439 m ²)		Roadway (848 m ²)	
11m ³		21m ³	
Vegetated Buffer Strip 50% TSS Removal		Vegetated Filter Strip 30% TSS Removal	
5.5m ³	5.5m ³	6m ³	15m ³
Enhanced Grass Swale 70% TSS Removal		Grass Swale 70% TSS Removal	
1.5m ³	9.5m ³	17m ³	4m ³

Figure 4-2: Quality Treatment Matrix

The proposed treatment train approach provides a total TSS removal of 82% for all three catchment areas. **Therefore, the SWM facility accompanied by the LID measures provides the necessary quality control across the site to satisfy the quality control objective outlined in Section 2.**

4.3 Conveyance

This Section provides an overview of the storm water conveyance through the development. Minor flows (<5-yr storm) will be conveyed within the roadside ditches and culvert network. Major flows (10- to 100-yr storms) will be safely conveyed within the right-of-way to the intended outfalls.

4.3.1 Roadside Ditches

As per the typical road cross-section these ditches will have a 1.0m bottom, 3:1 side slopes, an overall depth of 0.66m, and longitudinal slopes ranging from 1% to 2%. At the minimum slope, the ditches will contain a full flow capacity $2.949\text{m}^3/\text{s}$ which far exceeds the 100-yr inlet flow of $1.53\text{m}^3/\text{s}$ into the SWM facility.

4.3.2 Culvert Sizing

Culverts at specified locations within the site will need to be sized to ensure they contain adequate flow capacity. This analysis will be completed at time of detailed design.

4.3.3 Pond Outlet Channel

The pond outlet will discharge into a grassed swale to Matheson Drive ditch. The configuration of this channel is proposed to be a trapezoidal shape, 0.75m deep with a 0.5m wide bottom and 3:1 side slopes. With a longitudinal slope of 1.0% and Manning's n of 0.035 assigned for grass, the full flow capacity of the ditch is $3.16\text{m}^3/\text{s}$. In the case of emergency and the 100-yr flow of $1.533\text{m}^3/\text{s}$ spills out of the pond, the channel will have a flow depth of 0.55m leaving a freeboard depth of 0.20m to the top of channel.

4.3.4 Matheson Roadside Ditch

Re-ditching of Matheson Roadside ditch is proposed within the roadway to lower the elevation of the pond. The extent of this ditching is illustrated on the Concept SWM Facility drawing in **Appendix D**. The existing centerline of ditch will be maintained but lowered from the existing 1.5% slope to a 1.0% slope from house 1027 to the tie in location of the Pond Outlet Channel. This results in lowering the ditch at the tie in location by 1.02m overall.

5 EROSION AND SEDIMENT CONTROL

Erosion and sediment control measures are required during the construction phase to limit the amount of sediment leaving the site. Areas that are disturbed due to construction activity create potential for washing out of exposed areas resulting in blockages in downstream infrastructure such as culverts and ditches. To mitigate this exposure, erosion and sediment control measures should be installed and maintained throughout construction and until vegetation has been re-established.

These measures should be tailored to the final design of the development in order ensure proper protection. Therefore, these details will be prepared at the time of detailed design.

6 MAINTENANCE & OPERATIONS

The transfer of a SWM facility from the developer to the future owner shifts responsibility of the long-term operation and maintenance of the facility to the new owner. Similar to other municipal infrastructure such as sewers and treatment plants, the effective long-term operation of a SWM facility relies upon effective and consistent maintenance practices. Maintenance of the quality control facilities should be in accordance with Toronto Region Conservation Authority's *Inspection and Maintenance Guide for Stormwater Management Ponds and Constructed Wetlands (2018)* guide.

These measures should be tailored to the final design of the facility in order ensure proper maintenance & Operation. Therefore, these details will be prepared at the time of detailed design.

7 CONCLUSION

Monument Group (“Monument”) was retained by EFI Engineering Inc. to prepare a Preliminary Stormwater Management Report for the proposed Matheson & Rosedale Subdivision. The 23.54ha site is located in the Township of Montague situated in Lanark County and the jurisdiction of Rideau Valley Conservation Authority (RVCA). The intent of the stormwater management (SWM) design is to satisfy the following design objectives:

- 1) **Quantity Control** – The objective is to ensure that post-development peak flows do not exceed the pre-development levels for all storm events up to the 100-yr return period. This ensures that there are no negative impacts due to flooding on any lands downstream of the subject property.
- 2) **Quality Control** – Quality control target will be to meet a Level 1 Total Suspended Solids (TSS) removal efficiency at the *Enhanced* level of 80% long-term TSS removal rate, in accordance with the *MOE SWM Planning and Design Manual (2003)*.

The proposed development will consist of 41 detached single-family homes on 1 acre lots with a minimum frontage of 46m (150ft). Potable water will be drawn at each lot from private wells and disposed of in individual septic systems. The roadway will be a typical rural right-of-way, 20m wide, with a 7m driving surface, and 1.0m wide bottom ditches to convey runoff from the site to a stormwater management (SWM) facility.

The proposed SWM facility was selected as an extended detention wet pond to provide both quality and quantity control. The pond has been designed to provide Level 1 TSS removal efficiency equipped with a forebay and permanent pool to treat the 25mm – 4hr Chicago storm. For those areas of the development that cannot feasibly drain to the SWM facility, low impact devices such as vegetated buffers and enhanced grassed swales will provide Level 1 TSS removal efficiency as well. An active storage volume of 7656m³ within the pond will provide quantity control, sized to attenuate the 2 – 100yr return period events in the post-development conditions. The active storage will overcontrol flows, to ensure that pre-development release rates are met at each outlet location from the site. **Therefore, satisfying the design objectives identified above.**

Additional objectives such as a sediment and erosion control plan and owners’ operation and maintenance manual for the SWM facility will be prepared at detailed design.

Reviewed by:



Cody Oram, P.Eng.
Senior Project Manager

Submitted by:



Patrick Quinn, P.Eng.
Project Engineer

8 REFERENCES

Ministry of Environment (2003). *Stormwater Management Planning and Design Manual*.

Ministry of Transportation of Ontario (1997) *MTO Drainage Management Manual*

DRAFT – Stormwater Management Facility Design Guidelines & Standards (2012) *City of Ottawa Environmental Services Department, Surface Water Management Services Branch, Stormwater & Municipal Drainage Unit*

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Thornthwaite, C.W., and Mather, J.R., The Water Balance. *The Water Balance*. Publications in Climatology, vol 10, No. 3 Drexel Institute of Technology, Laboratory of Climatology, Centreton, New Jersey, 1055.

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Appendix A – Hydrology Methodology

This Section provides explanation on the hydrologic parameter selected for the each PCSWMM models.

Soil Conditions

Soil conditions for rural and undeveloped sites in Ontario are catalogued in the AG Ontario Soil Survey Complex by the Ministry of Agriculture, Food, and Rural Affairs and was used to determine the Hydrologic Soils Group (HSG) for the area. The site is comprised of Type B Farmington & Grenville Loam; moderately drained fine sandy loam and silt loam overlying loam till with moderate infiltration capacity.

Curve Number

Curve Number is a system developed by the U.S. Soil Conservation Service for estimating the volume of rainfall runoff in agricultural lands. Curve numbers are heavily dependent on the hydrologic soils group and land use type. A weighted curve number was calculated for the site by assigning curve numbers by land area based on Design Chart 1.09 in the MTO Drainage Management Manual (2008) for B soils with pastures and woodland forests.

Runoff Coefficient

Monument determined an arithmetic weighted runoff coefficient, R.C., to account for different land uses and soil types. Runoff coefficients were assigned based on Chart 1.07 in the MTO Drainage Management Manual (2008) HSG B (Loam) reflective of the overland slope. The runoff coefficient is used for calculating the time of concentration using the NashIUH method for the pre-development model.

Imperviousness

Imperviousness is a measure of surface hardness in a catchment area. Increased surface hardening due to building of water-resistant structures such as roads and rooftops reduces the infiltration capacity of ground surfaces and increases runoff. Imperviousness can be expressed as a percentage of hardened surfaces in the entire catchment area. The site imperviousness in post-development conditions was calculated for each catchment area.

Slope and Flow Length

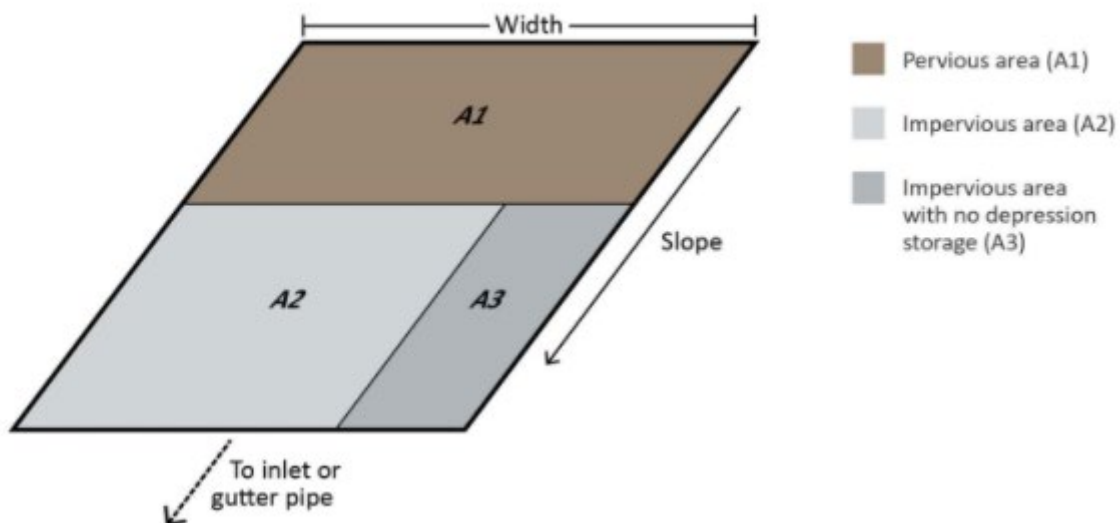
Slope and flow length influence the time it takes for runoff to reach an outlet. Flow lengths for each catchment were measured as the longest overland flow route to either the channelized route or outfall of the contributing area. Slope for each catchment was determined using the 85/10 method which is generally recommended for normal use.

Time of Concentration

The time of concentration describes the time required for water travel from the most remote point in a catchment area to the outfall. The time of concentration is used for the NashIUH method in PCSWMM and is calculated internally. The Airport method was selected in PCSWMM for calculating the time of concentration since the runoff coefficient for each catchment area is below 0.4.

Subarea Routing

Post-development catchments are divided into pervious and impervious subareas. Surface runoff can infiltrate in pervious surfaces represented by the curve number, whereas impervious areas will directly runoff. Overland flow is then generated from each subarea by approximating them as non-linear reservoirs (see image below).



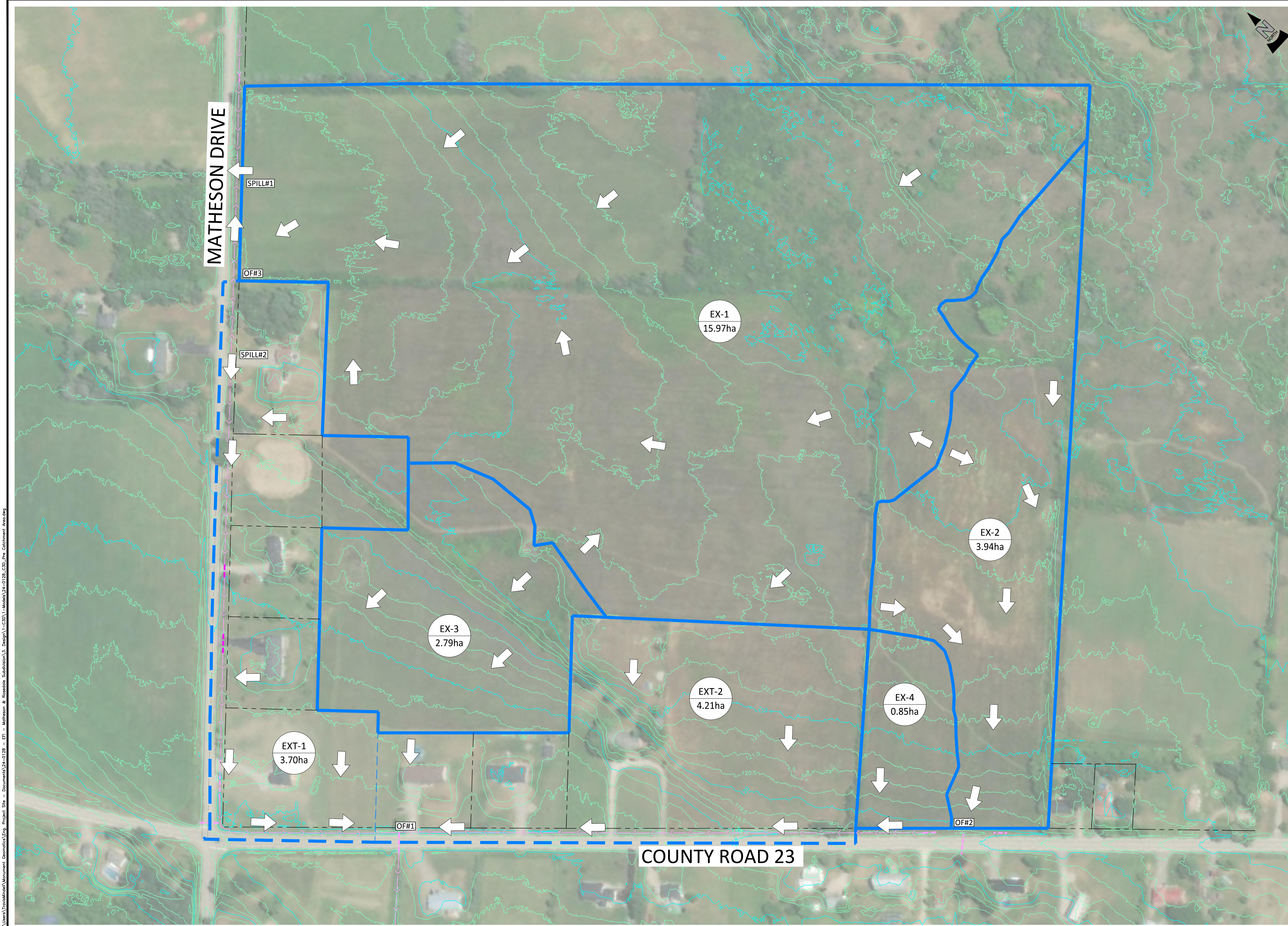
PCSWMM Support Excerpt describing Subarea Routing
(<https://support.chiwater.com/80217/subarea-routing>)

Typically, the overland flow from each subarea is independently routed to the outlet, however, PCSWMM allows the user to further subdivide runoff between subareas using the subarea routing tool. This creates internal routing between pervious and impervious surfaces. (e.g. roofs onto lawn surfaces). There are three selection options for the subarea routing tool:

- **IMPERV:** some percentage of the runoff from the pervious area is directed to the impervious area and then to outlet,
- **PERV:** some percentage of the runoff from the impervious area is directed on the pervious area and then to outlet,
- **OUTLET:** runoff from each subarea is routed directly to the outlet. (e.g. no subarea routing)

Monument selected to use the PERV command for the on-site catchments to route runoff from rooftops onto the grassed areas due to roof leader disconnection. This is represented by expressing a percent routed for each catchment (i.e. area of rooftop over the impervious area). The percent routed for each catchment are provided in the Hydrologic Input Parameters in **Appendix C**.

Appendix B – SWM Catchment Area



LEGEND

- PROPERTY LINES
- EX. DITCH
- EXISTING CULVERT
- 101 — CATCHMENT ID
1.0 — AREA (ha)
- CATCHMENT BOUNDARY
- EXTERNAL CATCHMENT BOUNDARY
- OVERLAND FLOW ARROW

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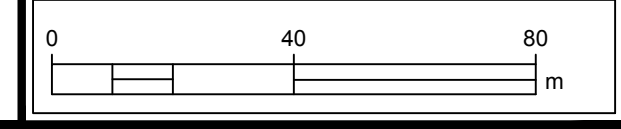


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DESIGNED: P.Q.	
CHECKED: C.O.	
ENGINEER: —	
DATE: 24/11/21	
H SCALE: 1:1250	
V SCALE: N/A	

SMART HOMES OTTAWA
 MATHESON AND ROSEDALE SUBDIVISION
 TOWNSHIP OF MONTAGUE

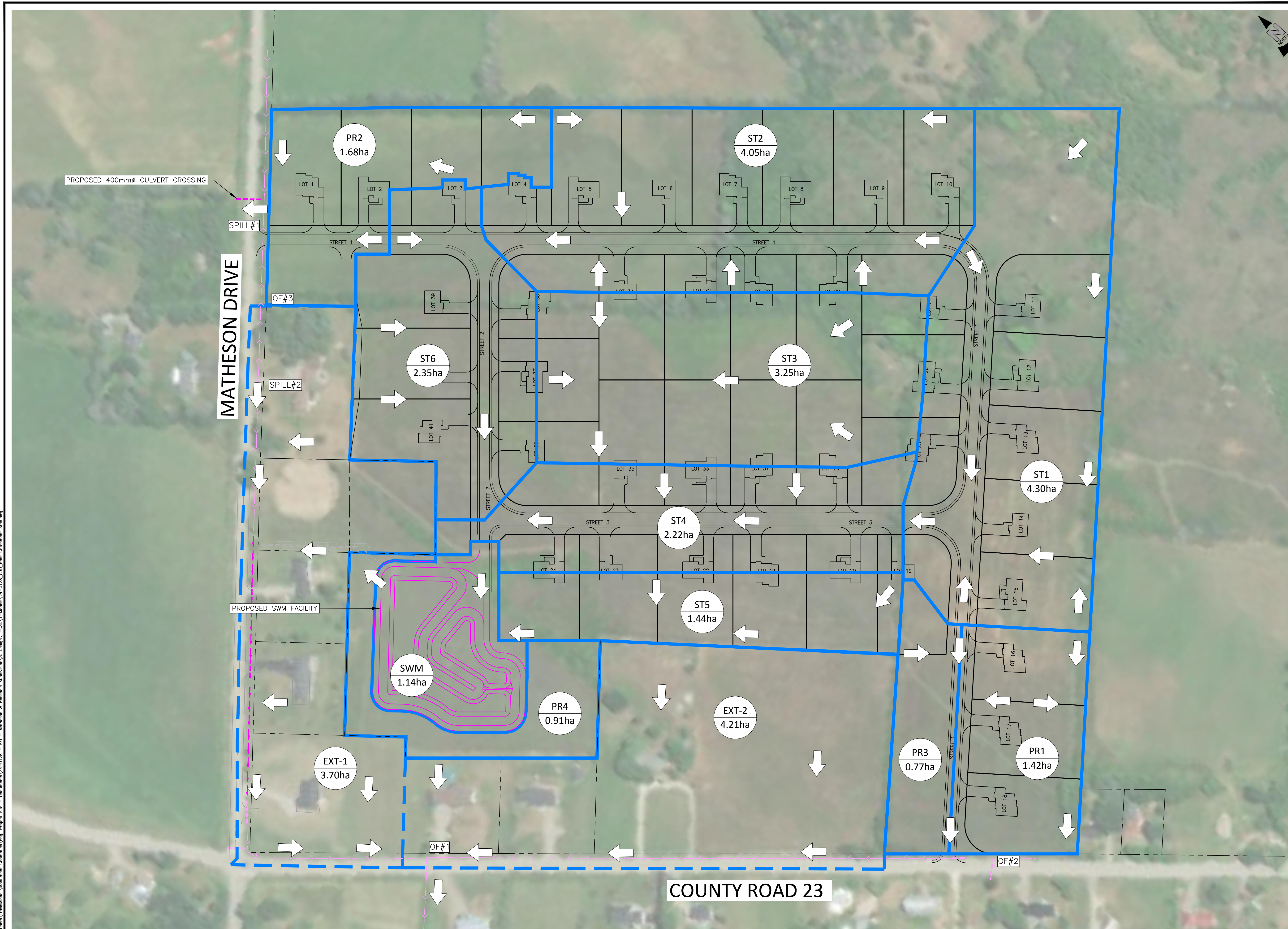
PRE-DEVELOPMENT STORM
 CATCHMENT AREAS

DRAWING No.
ST1



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LEGEND

- PROPERTY LINES
- - - EX. DITCH
- - - EXISTING CULVERT
- - - PROPOSED CULVERT

101 — CATCHMENT ID
1.0 — AREA (ha)

- [Blue Solid Line] CATCHMENT BOUNDARY
- [Blue Dashed Line] EXTERNAL CATCHMENT BOUNDARY
- [White Arrow] OVERLAND FLOW ARROW

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V SCALE: N/A	

SMART HOME OTTAWA
MATHESON AND ROSEDALE SUBDIVISION
TOWNSHIP OF MONTAGUE

POST-DEVELOPMENT STORM
CATCHMENT AREAS

DRAWING No.
ST2

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Appendix C – PCSWMM Hydrologic Input Parameters

Pre-Development

Catchment	EX-1	EX-2	EX-3	EX-4	EXT-1	EXT-2
Outlet Point	OF3	OF2	OF1	OF1	OF1	OF1
Area	15.97	3.94	2.79	0.85	3.70	4.21
Flow Length	516	425	195	137	168	143
Slope	0.5%	0.9%	2.9%	2.9%	3.4%	1.3%
Percent Impervious (%)	0.0%	0.0%	0.0%	0.0%	8.9%	10.5%
N Impervious	0.013	0.013	0.013	0.013	0.013	0.013
N Pervious	0.25	0.25	0.25	0.25	0.25	0.25
Subarea Routing (%)	-	-	-	-	75	72
Curve Number	71	74	74	74	65	65
Runoff Coefficient	0.33	0.35	0.35	0.35	-	-
Time of Concentration (min)	72	52	24	20	-	-

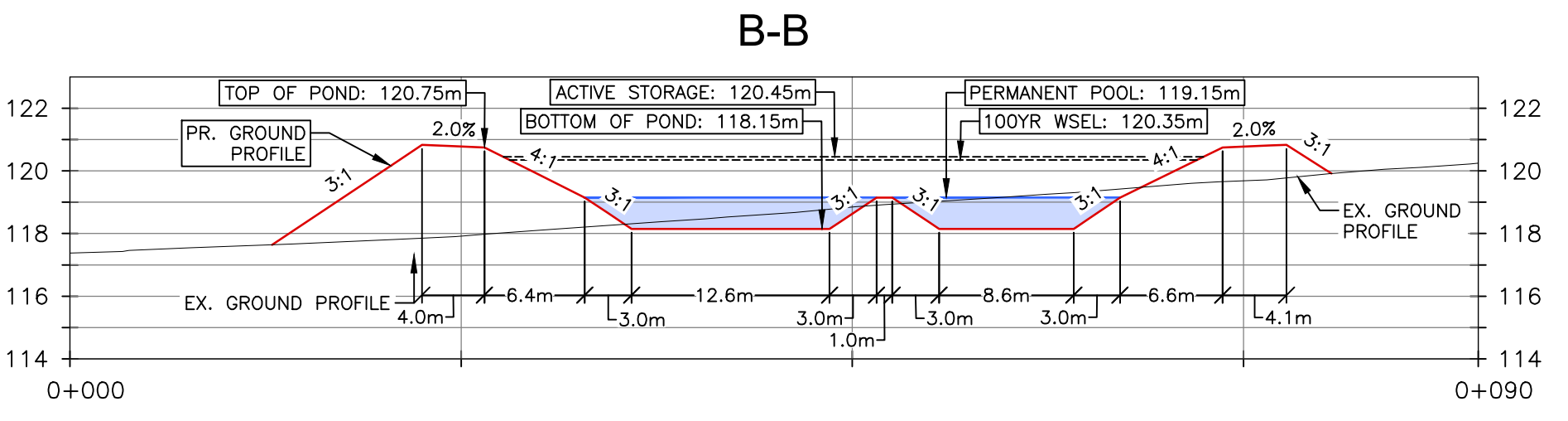
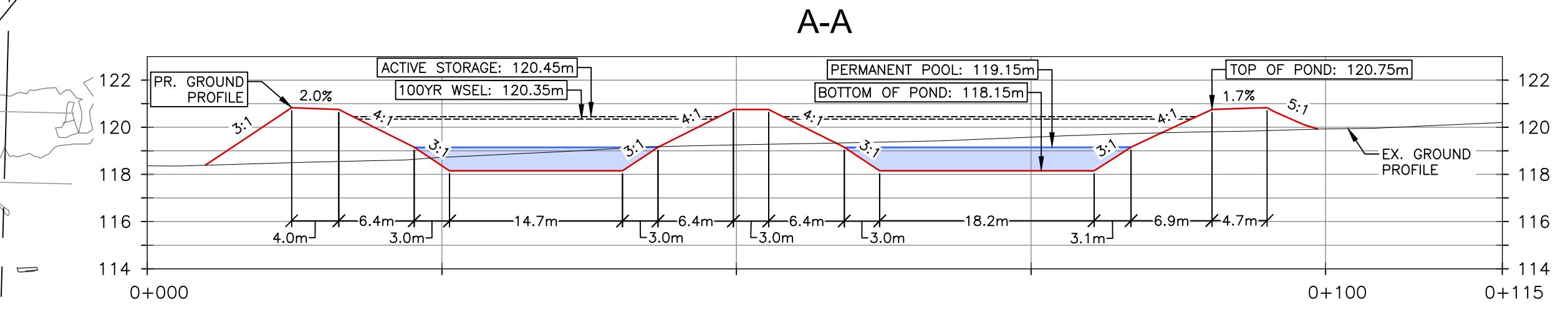
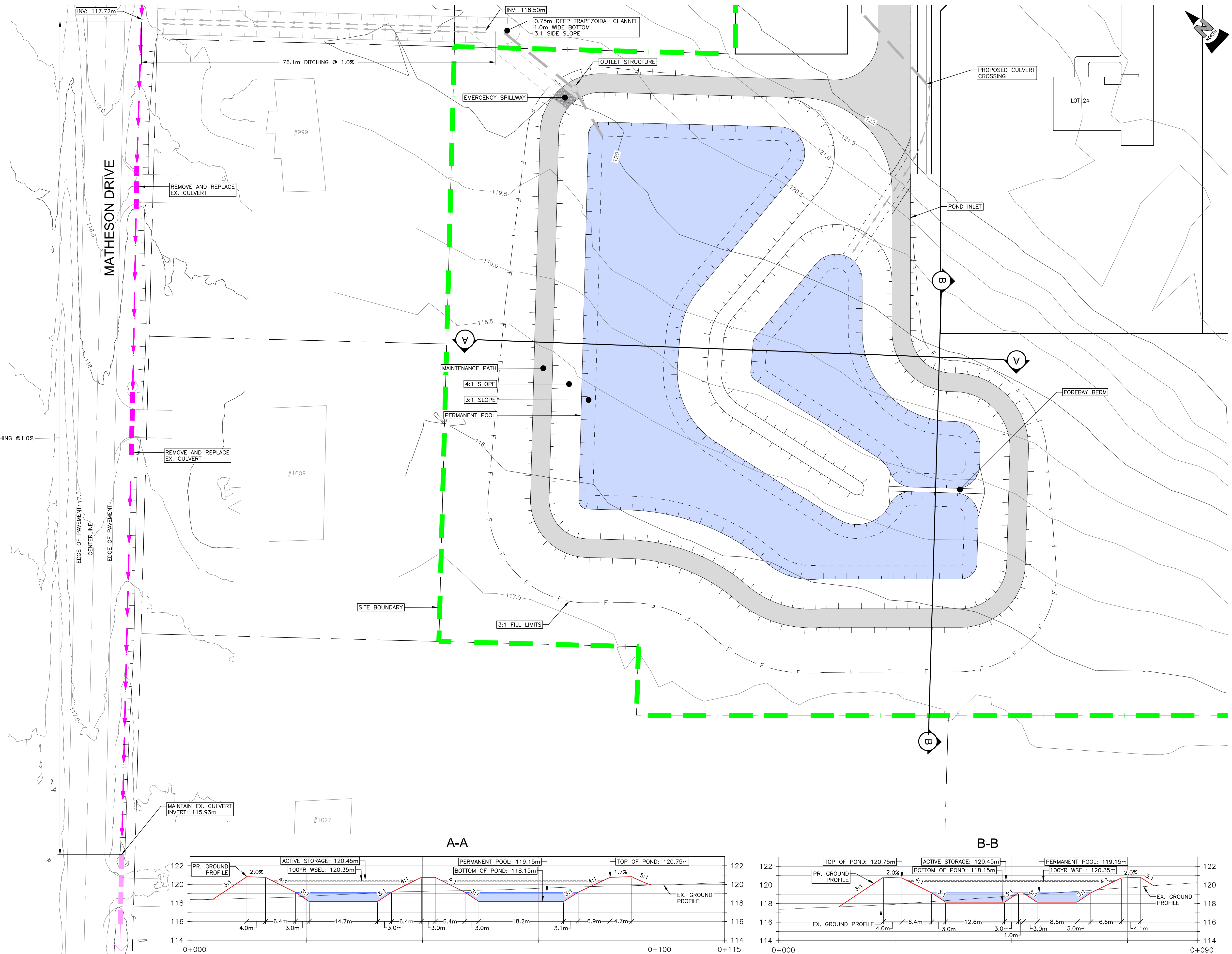
Post-Development

Catchment	PR-1	PR-2	PR-3	PR-4	EXT-1	EXT-2
Outlet Point	OF1	OF2	OF2	OF2	OF1	OF1
Area	1.42	1.68	0.77	0.91	3.703	4.21
Flow Length	45.38	24.64	78	84	168	143
Slope	2%	2%	2%	2%	3%	1%
Percent Impervious (%)	-	-	-	-	9%	10%
N Impervious	0.013	0.013	0.013	0.013	0.013	0.013
N Pervious	0.25	0.25	0.15	0.13	0.25	0.25
Subarea Routing (%)	15%	12%	-	-	75	72
Curve Number (AMCII)	77	77	60	77	65	65

Catchment	ST1	ST2	ST3	ST4	ST5	ST6	SWM
Outlet Point	ST4	ST6	ST4	SWM	SWM	SWM	OF1
Area	4.30	4.05	3.25	2.22	1.44	2.35	1.1385
Flow Length	81.53	67.14	67.62	38.67	40.10	59.99	180
Slope	1.5%	1.5%	1.5%	1.0%	1.0%	1.5%	0.5%
Percent Impervious (%)	17.0%	24.0%	6.0%	37.0%	6%	28%	52%
N Impervious	0.013	0.013	0.013	0.013	0.013	0.013	0.013
N Pervious	0.25	0.25	0.25	0.25	0.25	0.25	0.25
Subarea Routing (%)	55%	61%	100%	62%	100%	53%	100%
Curve Number (AMCII)	71	71	71	71	71	71	90

Appendix D – Concept SWM Facility

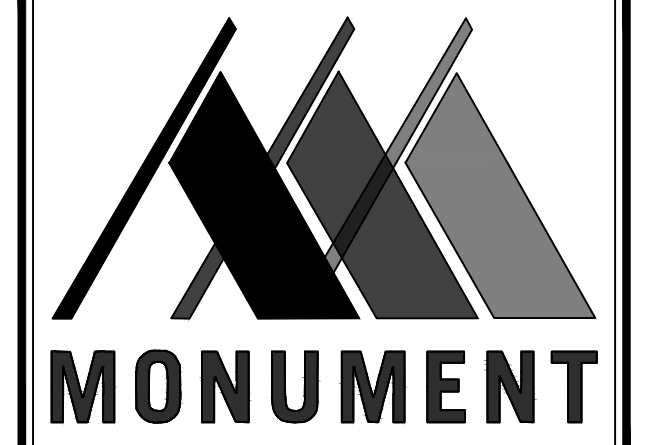
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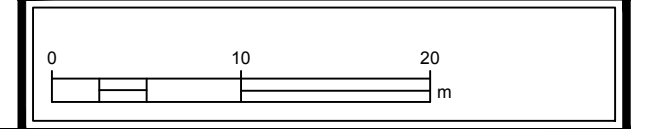
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SMART HOME OTTAWA
MATHESON AND ROSEDALE SUBDIVISION
TOWNSHIP OF MONTAGUE
STORMWATER MANAGEMENT POND
& CROSS SECTIONS

DRAWING No. **ST3**



Appendix E – Pond Sizing Stage-Storage-Discharge Sheet

Stage Storage Calculations

Matheson & Rose Dale Subdivision Wet Pond

Pond Details	
Bottom of Pond =	118.15
Permanent Pool Elevation (P.P) =	119.15
Extended Detention (E.D.) =	119.55
Active Storage (A.S.) =	120.45
Top of Berm Elevation (T.B.) =	120.75
Depth of Active Storage =	1.3
Depth of Permanent Pool =	1.0
Depth of Pond =	2.6
Pond Interval =	0.1

	Outlet 1 Orifice	Outlet 2 Sharp-Crested Weir	Outlet 3 Emergency Spillway
Invert =	119.15	119.85	120.45
Diameter =	0.15	0.40	5.00
C =	0.62		3.00 :1
Obvert =	119.30		
Radius =	0.08		
Area =	0.02		

Equations

Orifice = $Q = CA_o(2gh)^{0.5}$

V-Notch Weir = $Q = 1.38H^{2.5}$

Broad-Crested Weir = $Q = 1.67H^{3/2}$

Sharp-Crested Weir = $Q = 1.84H^{3/2}$

where,

C = Orifice Coefficient

A_o = Pipe Area, m²

h = Orifice Head, m

H = Head, m

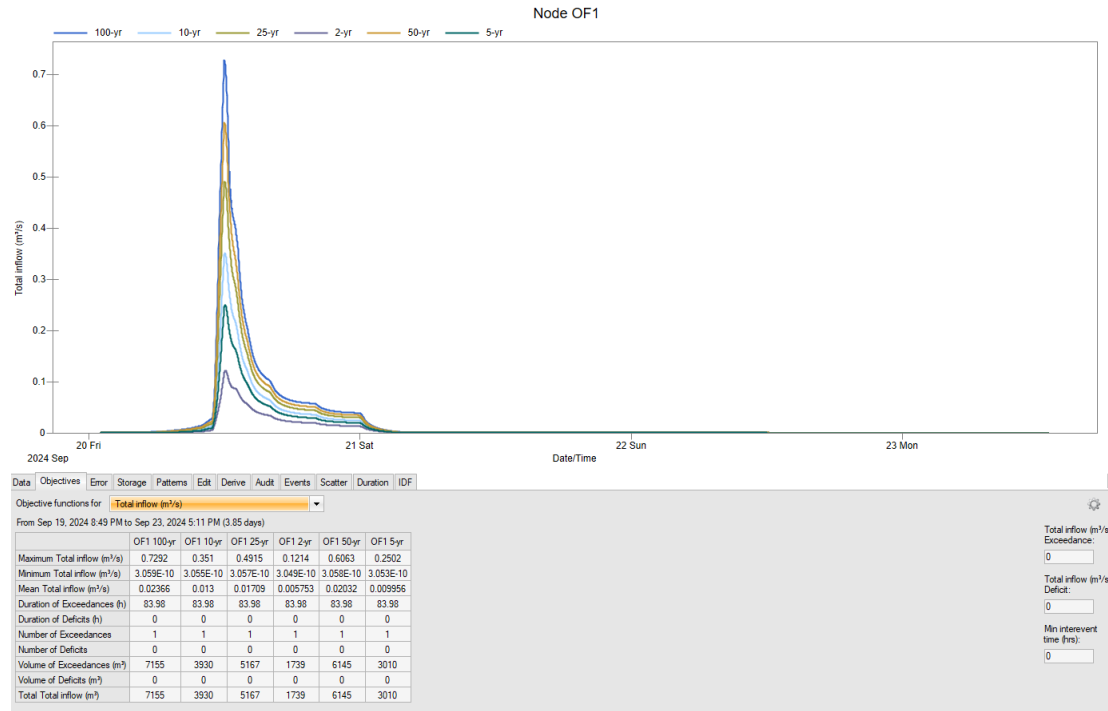
L = Length of Weir

	Outlet 1			Outlet 2			Outlet 3			PP Storage (m ³)	Active Storage (m ³)	Discharge (m ³ /s)		
	Orifice	Sharp-Crested Weir	Emergency Spillway	Orifice Head, h, (m)	Q _{orifice} (m ³ /s)	Head, H (m)	Q _{outlet 2} (m ³ /s)	Head, h (m)	Q _{outlet3}					
	Elevation (m)	Area (m ²)	Incremental Volume (m ³)	Cumulative Volume (m ³)	Active Storage Depth, (m)	Orifice Head, h, (m)	Q _{orifice} (m ³ /s)	Head, H (m)	Q _{outlet 2} (m ³ /s)	Head, h (m)	Q _{outlet3}	PP Storage (m ³)	Active Storage (m ³)	Discharge (m ³ /s)
	118.15	2994.21	0.0	0	0.00	0.000	0.000	0.00	0.000	0.00	0.000	0	0	0.000
	118.25	3141.23	306.8	306.8	0.00	0.000	0.000	0.00	0.000	0.00	0.000	307	0	0.000
	118.35	3290.48	321.6	628.4	0.00	0.000	0.000	0.00	0.000	0.00	0.000	628	0	0.000
	118.45	3440.87	336.6	964.9	0.00	0.000	0.000	0.00	0.000	0.00	0.000	965	0	0.000
	118.55	3592.40	351.7	1316.6	0.00	0.000	0.000	0.00	0.000	0.00	0.000	1317	0	0.000
	118.65	3744.98	366.9	1683.5	0.00	0.000	0.000	0.00	0.000	0.00	0.000	1683	0	0.000
	118.75	3898.75	382.2	2065.6	0.00	0.000	0.000	0.00	0.000	0.00	0.000	2066	0	0.000
	118.85	4053.67	397.6	2463.3	0.00	0.000	0.000	0.00	0.000	0.00	0.000	2463	0	0.000
	118.95	4209.71	413.2	2876.4	0.00	0.000	0.000	0.00	0.000	0.00	0.000	2876	0	0.000
	119.05	4366.94	428.8	3305.3	0.00	0.000	0.000	0.00	0.000	0.00	0.000	3305	0	0.000
P.P	119.15	4535.51	445.1	3750.4	0.00	0.000	0.000	0.00	0.000	0.00	0.000	3750	0	0.000
	119.25	4746.07	464.1	4214.5	0.10	0.100	0.005	0.00	0.000	0.00	0.000	0	464	0.005
	119.35	4956.95	485.2	4699.6	0.20	0.200	0.017	0.00	0.000	0.00	0.000	0	949	0.017
	119.45	5166.13	506.2	5205.8	0.30	0.300	0.023	0.00	0.000	0.00	0.000	0	1455	0.023
E.D	119.55	5370.29	526.8	5732.6	0.40	0.400	0.028	0.00	0.000	0.00	0.000	0	1982	0.028
	119.65	5575.42	547.3	6279.9	0.50	0.500	0.032	0.00	0.000	0.00	0.000	0	2529	0.032
	119.75	5781.55	567.8	6847.7	0.60	0.600	0.035	0.00	0.000	0.00	0.000	0	3097	0.035
	119.85	5988.66	588.5	7436.2	0.70	0.700	0.038	0.00	0.000	0.00	0.000	0	3686	0.038
	119.95	6196.78	609.3	8045.5	0.80	0.800	0.041	0.10	0.022	0.00	0.000	0	4295	0.063
	120.05	6405.89	630.1	8675.6	0.90	0.900	0.044	0.20	0.066	0.00	0.000	0	4925	0.110
	120.15	6616.00	651.1	9326.7	1.00	1.000	0.047	0.30	0.121	0.00	0.000	0	5576	0.168
	120.25	6827.13	672.2	9998.9	1.10	1.100	0.049	0.40	0.186	0.00	0.000	0	6249	0.235
	120.35	7039.26	693.3	10692.2	1.20	1.200	0.051	0.50	0.260	0.00	0.000	0	6942	0.312
A.S.	120.45	7252.38	714.6	11406.8	1.30	1.300	0.054	0.60	0.342	0.00	0.000	0	7656	0.396
	120.55	7466.49	735.9	12142.7	1.40	1.400	0.056	0.70	0.431	0.10	0.307	0	8392	0.794
	120.65	7681.61	757.4	12900.1	1.50	1.500	0.058	0.80	0.527	0.20	0.916	0	9150	1.501
T.B.	120.75	7897.48	779.0	13679.1	1.60	1.600	0.060	0.90	0.628	0.30	1.768	0	9929	2.457

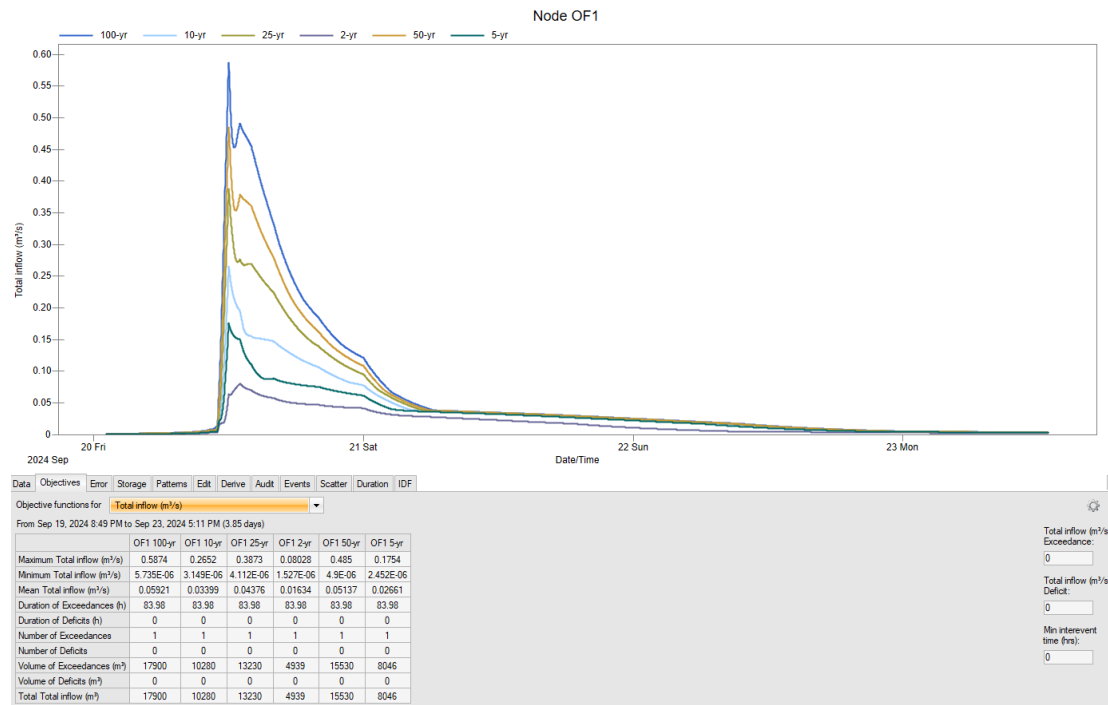
*Note: All units are in metric, unless otherwise stated

Appendix F – PCSWMM Model Outputs

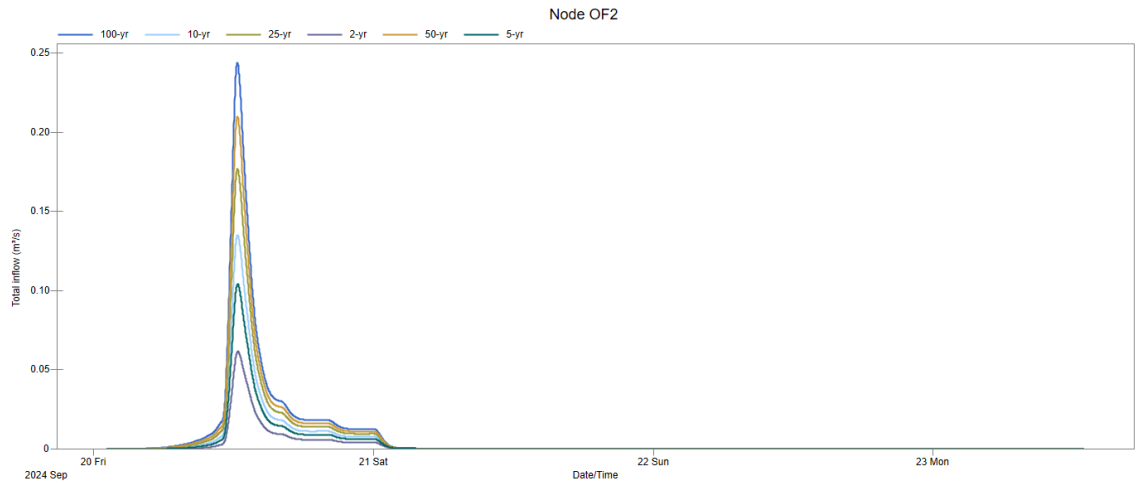
OF1 Pre-development



OF1 Post-development Controlled

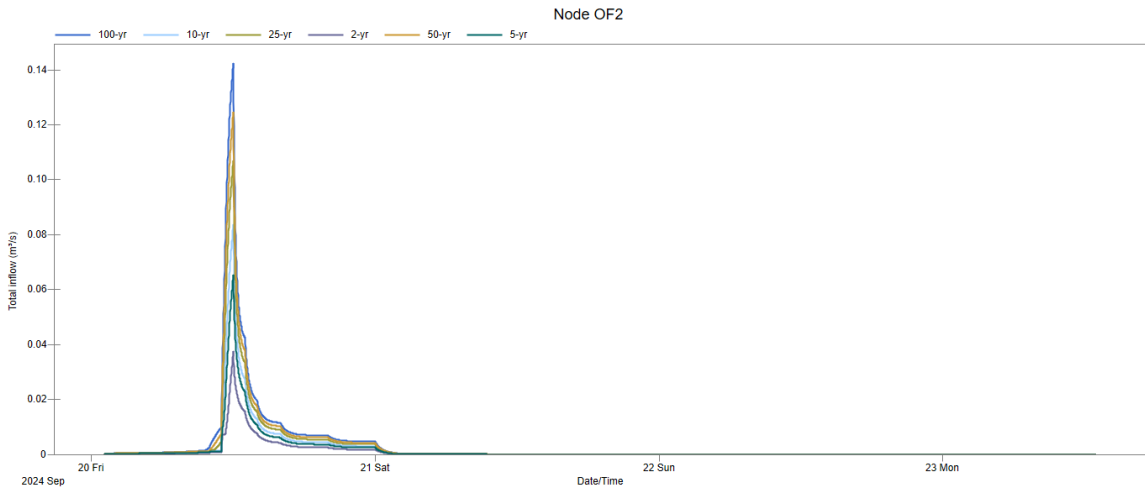


OF2 Pre-development



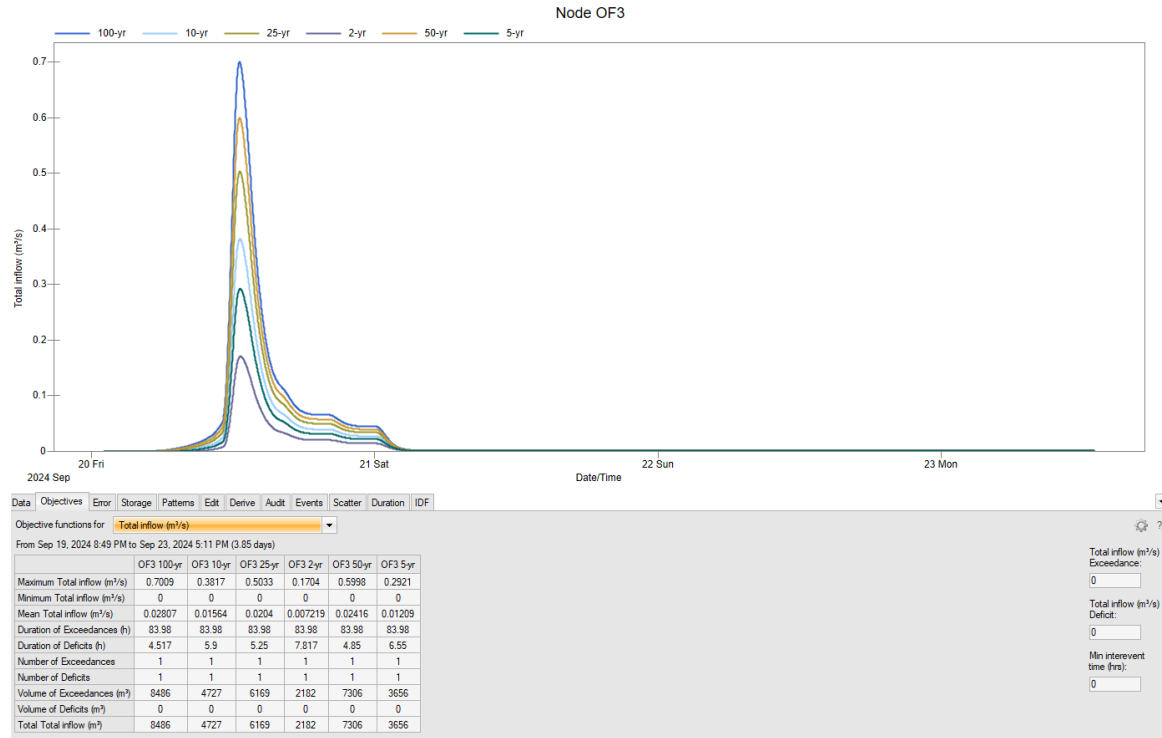
	OF2 100-yr	OF2 10-yr	OF2 25-yr	OF2 2-yr	OF2 50-yr	OF2 5-yr
Maximum Total Inflow (m³/s)	0.2439	0.1351	0.1769	0.06133	0.2098	0.104
Minimum Total Inflow (m³/s)	0	0	0	0	0	0
Mean Total Inflow (m³/s)	0.008027	0.004535	0.005882	0.002129	0.006937	0.003528
Duration of Exceedances (h)	83.98	83.98	83.98	83.98	83.98	83.98
Duration of Deficits (h)	60.83	62.07	61.47	63.97	61.12	62.67
Number of Exceedances	1	1	1	1	1	1
Number of Deficits	2	2	2	2	2	2
Volume of Exceedances (m³)	2427	1371	1778	643.6	2097	1067
Volume of Deficits (m³)	0	0	0	0	0	0
Total Total Inflow (m³)	2427	1371	1778	643.6	2097	1067

OF2 Post-development



	OF2 100-yr	OF2 10-yr	OF2 25-yr	OF2 2-yr	OF2 50-yr	OF2 5-yr
Maximum Total Inflow (m³/s)	0.1424	0.08364	0.1069	0.03737	0.1246	0.05526
Minimum Total Inflow (m³/s)	9.044E-11	9.037E-11	9.04E-11	9.027E-11	9.043E-11	9.034E-11
Mean Total Inflow (m³/s)	0.003281	0.001935	0.002459	0.0009612	0.002865	0.001535
Duration of Exceedances (h)	83.98	83.98	83.98	83.98	83.98	83.98
Duration of Deficits (h)	0	0	0	0	0	0
Number of Exceedances	1	1	1	1	1	1
Number of Deficits	0	0	0	0	0	0
Volume of Exceedances (m³)	991.8	585	743.5	290.6	866.3	464
Volume of Deficits (m³)	0	0	0	0	0	0
Total Total Inflow (m³)	991.8	585	743.5	290.6	866.3	464

OF3 Pre-development



OF3 Post-development

